P 778

THE NEW IRRIGATION ERA

Published by the Irrigation Branch of the Madras Public Works Department, Madras-5, South India.

(For Departmental Circulation only)

" I believe in Hands that work,
Brains that think and
Hearts that love. "
—Elbert Hubbard.

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We Wish You

A Happy and Prosperous

New Year.

WELCOME ADDRESS ON THE EVE OF THE OPENING OF THE ALIYAR DAM ON 2nd OCTOBER 1962

By

* SRI U. ANANDA RAO, B.E., M.I.E., (Ind.), M.Am. Soc. C.E., I.S.E.,

Honourable Chief Minister, Honourable Minister for Works, Workers of Parambikulam-Aliyar Project, Ladies and Gentlemen,

On behalf of the officers and staff of the Parambikulam-Aliyar Project, it is my proud privilege to welcome you all here today on the occasion of the opening of the Aliyar Dam. It is in the fitness of things that our Chief Minister, the friend of the poor and needy, should inaugurate, on this auspicious day, Gandhi Jayanthi, the opening of the Aliyar Dam, the first to be completed in the series of eight Dams under the Parambikulam-Aliyar Project. This is another landmark in the history of Irrigation of Coimbatore district, being the fourth major Irrigation Project undertaken by the National Government in this district after the attainment of Independence, the first three being the Lower Bhavani Project, the Mettur Canal Scheme and the Amaravathi Reservoir Project. We are indeed very happy and thankful to the Chief Minister, Minister for Works and other distinguished guests for participating in this function in spite of their multifarious other duties.

2. This Parambikulam-Aliyar Project envisages the utilisation of a few streams in the Anamalais which traverse through both the States of Madras and Kerala and ultimately fall into the Arabian Sea. These rivers have thus become inter-State and the project has been so designed as to serve the interests of both the States. The Project, in brief contemplates integration of seven rivers, five on the Anamalai Hills and two on the plains by constructing reservoirs across them and inter-connecting them through tunnels. These tunnels carry water thus impounded in the reservoirs to lower levels and ultimately to the plains of Coimbatore District. Parambikulam represents the former group of five rivers, viz., Nirar, Sholiar, Parambikulam, Tunacadavu and Tekkadi and the latter, two rivers Aliyar and Palar. These rivers lie at various elevations ranging between plus 3,760 and plus 1,050, which incidentally enable us to utilise the drops between the rivers and develop power. The Project provides irrigation to an extent of 2,40,000 acres in Coimbatore district and an additional power to Madras grid with an installed capacity of 1,80,000 kilo watts from 5 power-houses. The existing ayacut under Aliyar below this dam, of about 6,400 acres in Madras State and 20·000 acres including 5,000 acres under Vardhuvida in Kerala State, get stabilised irrigation by the construction of this reservoir.

3. The project contemplates six major dams and two minor dams, 12 miles of inter-connecting tunnels, 30 miles of lined contour canal along the rugged hill slopes, 146 miles of lined canals and hundreds of miles of distributaries and field channels, etc. The first in the series is the one across Nirar, from where water will be diverted to Sholiar through a tunnel 14,500 feet long. Sholiar will have a reservoir with a dam, about 350 feet high just above Kerala State limits from which a tunnel 3,800 feet long takes off to divert water to the lower Parambikulam Reservoir. The difference in levels of these two reservoirs is about 1,300 feet which will be utilised to develop power to an extent of 70,000 kilo watts. Sholiar Dam diverts water not only to Madras but also to Kerala, whose power requirements at 380 cusecs are being ensured even during bad years. Parambikulam Reservoir will be the largest reservoir in the series whose gross capacity will be 16·800 million cubic feet and net capacity 9,000 million cubic feet. A tunnel 7,500 feet long conveys water from Parambikulam Reservoir to Tunacadavu Reservoir. Tunacadavu dam will be an earthen dam

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70 feet high. It will be a balancing reservoir and receives regulated supplies from Parambikulam Reservoir on the left flank and Tekkadi river on the right. From Tunacadavu Reservoir, water is taken through an approach channel and a tunnel 13,000 feet long to the Madras plains. A fall of about 300 feet at the end of the tunnel is being utilised for developing power to an extent of 30,000 kilo watts at Sirkarpathi Power House. The tail water of this power house is taken along two canals to feed Aliyar and Tirumurthi Reservoirs and another canal Sethumadai canal to irrigate about 3,756 acres in Pollachi taluk. One of the canals, called the Contour Canal, is taken along difficult hilly terrain on the northern slopes of Anamalai Hills, tunnelling through ridges and crossing valleys and dales over aqueducts and superpassages. Ultimately the Contour Canal falls into Tirumurthi Reservoir, in Udumalpet taluk. Aliyar and Tirumurthi Reservoirs, located in the plains of Madras are intended to receive and store waters for distribution amongst various canals taking off from them for catering to the needs of 2,40,000 acres in Pollachi, Udumalpet, Dharapuram and Palladam taluks of Coimbatore district. Pollachi and Vettaikaranpudur canals take off from Aliyar Reservoir and Parambikulam Main canal and Udumalpet canal from Tirumurthi dam.

4. The Aliyar dam across the Aliyar which the Honourable Chief Minister is going to open today is about 10,500 feet in length, of which about 2,990 feet is of masonry and the rest of earth. It is designed for a gross capacity of 3,860 million cubic feet at Full Reservoir Level plus 1,050.00. Three innovations have been introduced in the design of this dam. Firstly, provision of river sluices in a tunnel bored through the rocky hill adjacent to the river, secondly, the river is plugged with an earthen dam instead of a conventional masonry dam or spillway and thirdly, location of spillway over a rocky hill instead of across the river. The height of earthen dam is 142 feet above lowest foundations which is a record height for South India. From this reservoir, two irrigation canals take off, called Vettaikaranpudur and Pollachi canals.

5. The cost of this dam is roughly Rs. 229 lakhs out of Rs. 35.08 crores of the whole Parambikulam-Aliyar Project. The earthen dam construction was started in January 1960 and about 1,01,000 units or 101 million cubic feet of earthwork was completed in a record time before the end of June 1962. This quantity of earth will be sufficient to lay a road 40 feet wide 3 feet high and 160 miles long. This was possible by using the heavy earth-moving machinery like 'CATERPILLAR' DW 20 Scrapers, D8 Pushers and D7 Levellers, etc. In the initial stages, old Scrapers which had served their full life practically in other completed projects were put on this work but due to long usage and deteriorated condition they gave way frequently, and so much out-turn could not be obtained from these old scrapers. Subsequently one fleet of eleven new DW 20 Scrapers was purchased and put on this work on 16th August 1961. Though these gigantic earth-moving machines are partly responsible for the early completion of this earth dam, work could not have been completed but for the willing co-operation and hard work turned out by the specially trained operators and mechanics. But for their sustained work as a team, both during day and night, it would not have been possible for completing the earth dam in so short a period.

6. The masonry dam portion involving about 44,000 units of random rubble masonry in cement mortar was taken up in February 1960 and completed by about July 1962. In the execution of this dam, the broader interests of the working masses have always been kept in mind. Most of the masonry work was done on job work system, thereby eliminating the middleman's profit.

7. Though about 36,000 acres in Pollachi taluk are to be irrigated by this reservoir through Vettaikaranpudur and Pollachi canals finally, it is proposed to limit supplies to about 7,500 acres only under the two canals this year. Irrigation will be extended progressively in the coming years to the remaining areas also, as and when new reservoirs are constructed. This is one of the noteworthy aspects of this project, where the

lands are brought under beneficial use progressively without waiting for the completion of the entire project. Irrigation under the project will be generally limited to dry crops only, but about 20 per cent of the area will be permitted to raise paddy to cover seepage areas and other areas which are, *primafacie*, unfit for dry cultivation.

8. One more special feature of this project is that all the canals, distributaries and field channels up to 50 acres limit will be lined with concrete to prevent seepage losses. This enables very economical use of water and it is thereby made possible to give benefit to an additional area of about 40,000 acres. Bothies between 50 and 25 acres limits will be excavated at Government cost and those below 25 acres limit will have to be excavated by the ryots.

9. I have to take this opportunity to thank the officers and staff of the Revenue, Electricity, Forest and other Departments who had worked in close co-operation with the Public Works Department and the thousands of workers who had toiled ceaselessly to put the waters of Aliyar into use for the prosperity of this area and thereby contributing to the general rise of the standard of living. Naturally, the brunt of the work had fallen on the shoulders of the Public Works Department. Every one in the project had worked with a spirit of dedication and I have to thank personally one and all for their sustained and loyal work. It is very difficult for me to single out any officer as everyone had done his allotted work.

10. Our thanks are also due to the skilled drivers and mechanics who have done their best in running the heavy earthmoving machinery and several job workers, who did the work most economically and also to several firms, Messrs. Larson and Toubro, Burma-Shell, Standard Vacuum Oil Company Limited, Caltex, Cement Marketing Company and others for their co-operation.

11. In all these large undertakings, specially meant for the welfare of people the hand of God is always there. Without God's blessings nothing could have been achieved and I pray to Him that He may continue to bless the Project for early completion and to provide it with never failing supplies of water and make this part of the country run with proverbial milk and honey.

12. With these words, I request the Minister for Works to request the Chief Minister to inaugurate the Reservoir by throwing open the sluice gates for irrigation, thus ushering in an era of prosperity.

Salient Features of Aliyar Dam.

1. Deepest foundation level in river bed	+	914.50
2. Average bed level of deep course ..	+	918.00
3. Sill of river sluice	+	930.00
4. Crest of spillway	+	1,040.00
5. F. R. L. and M. W. L.	+	1,050.00
6. Road bridge level over spillway ..	+	1,059.00
7. Width of roadway over dam between kerbs.		14′
8. Spillway vents—11 Nos. with radial shutters.		30′x10′
9. Maximum flood discharge		40,957 cusecs.
10. Capacity at F. R. L.		3,684 m.cft.
11. Total quantity of masonry ..		44,000 units. of 100 cft.
12. Total quantity of earth work for earth dam.		1,01,000 units. of 1,000 cft.
13. Length of Masonry Dam.		2,990'
14. Length of Earth Dam		7,510'
15. Total length of Dam		10,500'

Salient Features of Parambikulam-Aliyar Project.

Number of dams		8
Total length of tunnels		12 miles.
Length of main canals		146 miles.
Ayacut of the Project		2,40,000 acres.
	Dry	1,92,000 „
	Wet.	48,000 „
Total annual yield		24,000 tons of rice 38,400 tons of millets.
Number of Power Houses.. ..		5
Power installation		1,80,000 K.W.
Firm energy		70,000 K.W.
Total cost		Rs. 35.08 crores.
Year of commencement		April 1959.
Year of partial benefit		October 1962.
Completion of Project		October 1966.

RESEARCH WITH CENTRAL FUNDS—A REVIEW

By

* Sri J. WALTER, B.A.,B.E., M. Irrn. Eng. (Colorado).

Synopsis.

The various Central Government bodies like the Central Board of Irrigation and Power, Council of Scientific and Industrial Research, National Buildings Organisation, etc., finance various research schemes undertaken by the Concrete and Soil Research Laboratory at Madras and the Irrigation Research Station at Poondi. The scope of this article is to familiarise the scientists and the engineers of our State with the centrally sponsored research activities conducted in our Public Works Department Laboratories.

Introduction.

The first Irrigation Research Station in Madras State was established in the year 1944 at Poondi, 37 miles away from Madras. In the course of its existence, roughly over a period of two decades, this station has built up from humble beginnings a record of usefulness and has been acknowledged as one of the foremost hydraulic research institutes in the country by virtue of its contribution to various problems. The Concrete and Soils Laboratory made its beginning as a Soil Mechanics wing in the Poondi Irrigation Research Station in the year 1945. This section was eventually moved over to Madras as a separate section in June 1946 as the Physics and Soil Mechanics Laboratory and was housed in two small rooms attached to the Chief Engineer's Office, Chepauk. The Laboratory moved into its own buildings in December 1947 under the name of Soil Engineering Research Station. The Concrete Laboratory formed subsequently in December 1947 in an adjacent building was later amalgamated with the Soil Engineering Laboratory. The Laboratory has since then been functioning as "The Concrete and Soil Engineering Research Laboratory" as a single research unit in the Chepauk Public Works Department Office premises adjacent to the Presidency College and in rear of the new imposing architectural building for the Public Works Department Offices.

The Central Board of Irrigation and Power as a body interested in the development of research in the fields of irrigation and power has allotted problems to research institutions all over the country. The Concrete and Soil Research Laboratory at Chepauk and the Irrigation Research Station, Poondi have been allotted some useful problems on Fundamental and Basic Research, the variety and scope of which are detailed below:—

CENTRAL BOARD OF IRRIGATION AND POWER SCHEMES: AT SOIL MECHANICS AND CONCRETE LABORATORY, MADRAS.

1. Engineering properties of soils.

Under this heading the following subjects are being studied:—

1 Correlation between mechanical and mineralogical composition of soils and their Engineering properties.

2. In situ tests for the determination of soil properties such as soil modulus, permeability, etc.

3. Swelling pressures.

A large number of samples were collected from the various parts of Madras State and the correlation between the liquid limit and plasticity index for all the samples were obtained and based on this, the soils of Madras State were classified into five distinct types and a soil

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map showing broadly the places where such soils are met with has been prepared. Successful correlations between the following Engineering properties were arrived at based on tests on a large number of samples :—

1. Compressive strength of soil samples in the undisturbed state with Dry Density and clay content.

2. Dry density of soil at optimum compaction with percentage smaller than 40 sieve and the liquid limit of the fraction passing 40 sieve.

3. Dry density of soil at optimum compaction with shrinkage ratio, shrinkage limit, per cent passing 4 sieve, per cent passing 40 sieve and the plasticity index.

4. Liquid limit with clay content and the mineralogical composition of the clay fraction as expressed by Silica sesqui-oxide ratio.

5. Volumetric shrinkage in the clay content and silica sesqui-oxide.

With reference to in situ tests, field load bearing tests have been conducted at a large number of sites and correlations between field and laboratory values of load bearing capacity worked out. Based on a correlation obtained between the settlement of the loaded plate at a particular load and the safe bearing capacity, a simplified procedure for determination of bearing capacity has been evolved. Field determination of soil modulus, permeability and shear strength were carried out and these were compared with the corresponding laboratory values. Under swelling pressure an improved device for a more rapid and more accurate determination of swelling pressures has been developed. Field studies on ground movements and settlement of structures were carried out.

2. Sub-soil flow.

In Dr. Khosla's theory on Design of Weirs on permeable foundations, infinite depth of pervious strata is assumed. In actual practice, in our State the pervious strata are, in many instances of a finite depth comparable to the width of the structure. It was noticed that in such cases, the uplift pressure obtained by electrical analogy experiments were not in agreement with Dr. Khosla's theory. Hence a systematic study of the theory of Design of weirs on limited depths of pervious stratum has been taken up in the Laboratory. The study is being carried out with electrical models. Theoretical investigations are also under way. It is hoped that when the study is over, a set of useful tables would be ready on Design of Weirs on permeable foundations of finite depth.

3. Standardisation of the use of surki and other Pozzolanic Materials in Mortar and concrete.

Under the above study, detailed tests are carried out on various pozzolanic materials like surki, flyash, coalash, etc., to investigate the following specific problems:—

(*a*) the determination of suitable indices for pozzolanic activity; and

(*b*) standardisation of pozzolanic materials and the condition under which they may be used.

In this connection detailed tests are conducted for chemical and physical properties of the pozzolanas in addition to strength tests on mortars and concrete by incorporating the pozzolana in them.

The results of the tests so far conducted on surki samples have indicated that for every soil there is an optimum temperature of burning for getting maximum pozzalanic activity and this optimum temperature range falls between 600°C and 800°C. From the results it has also been observed that the pozzolanic activity of surki increases with increase in fineness of the surki and as the temperature of burning increases there is reduction in the fineness of surki obtained.

The results of the tests conducted on Basin Bridge Flyash indicate the pozzolanic properties of the ash and from the results it is seen that it can be used as replacement in construction works up to about 15 per cent economically and very safely. Tests on Neyveli flyash are underway

4. Principles of Mortar and Concrete Mix Designs.

The following are the items of work that are being studied:—

(*a*) To investigate and arrive at suitable methods of designing mortar and concrete mixes.

(*b*) Basic research in concrete mix design in relation to thermal properties. permeability and dynamic modulus of rigidity.

(*c*) To investigate the availability of materials for mortar and concrete (i) aggregates, (ii) cement and hydraulic limes and (iii) air entraining agents.

(*d*) To study alkali reactivity and petrographic characteristics of aggregates.

(*e*) Investigations relating to the strength of masonry and concrete.

By the method of trial mixes, suitable mixes for materials available in and around Madras City have been designed. An indigenous material called Sapindus Emarginatus (Soapnut) is being investigated for its qualities as an air entraining agent in concrete. A detailed study on random rubble masonry in flexure is also underway.

5. Experimental Methods of stress analysis in Hydraulic Structures.

Under this problem, the following works are to be taken up:—

(*a*) The technique of casting synthetic resins like araldite to be finalised for casting sheets.

(*b*) Analysis of two dimensional models with different types of openings, evaluation of P and Q inside the model by finding (P—Q) by Fringe pattern and (P+Q) by the use of extensometers. electrical analogy and relaxation methods, comparison by using other techniques such as membrane methods.

(*c*) Assembling and fabrication of 3-D Models. equipment by freezing Technique for studying 3-D Models. extending the study of complicated models including different types of openings in dams and other hydraulic structures and analysis and evaluation of stresses in different sections by different methods and their comparison.

(*d*) Comparison of the results with stresses in prototypes from the readings of strain-meters, Joint meters, etc., embedded in Hydraulic structures.

However formal sanction of staff has not yet been received from the Government.

The Irrigation Research Station at Poondi is investigating the following problems.

6. Minimising losses due to absorption, percolation and evaporation in still and flowing water in irrigation.

The Buderi tank near the Research Station has been chosen for conducting tests. Originally cetyl alcohol was applied in the form of a solution but now by the emulsion method. a more economical application is being tried. The cetyl alcohol when thrown on the water surface disperses to form a monomolecular film over the water surface which minimises evaporation. The above experiments have indicated that about 20 per cent savings can be achieved. Sufficient staff has not yet been utilised for want of formal sanction by Government.

7. Sedimentation studies in Lower Bhavani Project.

The silting characteristics of the Lower Bhavani Reservoir and the silt load contributed by the two rivers Moyar and Bhavani are being studied. The equipment required for the studies such as silt samples and other laboratory equipment wire rope. trestles, winch, echo-sounder, etc., for the cradle gauging stations have been procured.

The design for the two cradle gauging stations has been finalised and the work on the erection of the cradle gauging stations is in progress. The range points defining the range lines across the reservoirs along which soundings are to be

taken to determine the silt depth are fixed at the maximum water level contour.

8. Minimising loss of head through improvement of Trash Rack, Penstock Gates, Spiral Chambers, Draft tubes, Downstream Flows and Silt Disposal Technique in Hydro-Electric structures.

The determination of the loss of head taking place in the Hydroelectric installations by adopting the different types of trash racks, penstock gates, spiral chambers, etc., now in vogue will be first investigated by model studies.

After determining the loss of head as mentioned above, the possible improvements consistent with structural stability will be carried out to the various components mentioned above and the effect of these improvements will be studied.

The results of the studies will be analysed in the form of charts in dimensionless parameters, wherever possible so that they may be of universal application. The work is held up for want of formal sanction of staff by Government.

Council of Scientific and Industrial Research Schemes.

Schemes in Concrete and Soil Research Laboratory—Madras.

9. Studies on the corrosive constituents of soils in Madras State.

Laboratory investigations comprising of physical. chemical and electro-chemical tests on soil samples, accelerated corrosion tests in different corrosive media and field studies comprising of burial tests, soil resistivity survey, potential survey and current drainage survey are being conducted to see how far the relative corrosivity of a soil can be assessed from a study of its corrosive constitutents.

So far about 700 soils have been studied and the results of polarisation measurements have been encouraging. Studies are in progress to assess the possibilities of cathodic protection as a method of protection underground.

10. Investigation of the correlation between Atterberg Limit and Compression Index and Swelling Index of Soils.

Correlations between Atterberg limits and consolidation characteristics of soils of Madras State are being attempted by statistical analysis. Correlations between liquid limit versus compressive and swelling indices and plastic index versus compression indices are obtained separately for undisturbed and remoulded soils.

About 400 samples have been tested so far. All the correlations obtained are found to be highly significant with correlation co-efficients more than 0.7 and levels of significance more than 99 per cent.

11. Conducting artificial rain making experiments at Tiruchirappalli in Madras State.

Artificial rain making trials have commenced at Tiruchirappalli and an attempt is being made to scientifically study the seeding of rain bearing clouds and analyse the results statistically in a vigorous and scientific manner. Hydroscopic nuclei are being sent up in the air from ground generators from two centres at Tiruchirappalli. Seeding is being done by adopting random numbers, taking into account the Meteorological factors at Tiruchirappalli. Rainfall data in the target and control sectors are being analysed to study the results of these seeding trials.

12. Mortar making properties of Ennore sand.

The white sand at Ennore is the Indian Standard Sand used in the testing and standardisation of cements. An investigation is being made to see whether any improvements can be effected so that better strength characteristics can be obtained by using different fractions of this sand to satisfy the requirements of specifications of a standard sand with respect to yield and economical feasibility.

SCHEMES AT POONDI RESEARCH STATION.

13. Design of high head gates.

Investigation on down pull and vibration characteristics of a slide gate for different conditions of heads, gate openings, lip shapes, flow patterns and air supply is made with the help of locally fabricated instruments and electronic equipments. Further studies are programmed for analysing the existing and damping forces of vibrations for mitigating the vibrations, designing proper vent for preventing cavitation and also for carrying out prototype research work on gates for confirmation of model results.

NATIONAL BUILDINGS ORGANISATION SCHEME.

14. Building mortars and strength of brick masonry.

The scheme at Soil Mechanics and Concrete Laboratories, Madras, consists in testing of brick masonry cubes of size 18 inches with two different types of bricks (chamber burnt stock bricks and chamber burnt country bricks) having a compressive strength of more than 1,000 psi with 8 different mortars and cured for 2 sets of different ages as indicated below:—

1.	1 : ¼ : 4 ..	Cement : Lime : Sand	7 days and 6 months.
2.	1 : 6 ..	Cement : Sand	14 days and 6 months.
3.	1 : 1 : 2	Lime : Surki : Sand	14 days and 6 months.
4.	1 : 2 ..	Lime : Surki	14 days and 6 months.
5.	1 : 3 ..	Lime : Surki	28 days and 6 months.
6.	1 : 1 : 2 ..	Lime : cinder : Sand	14 days and 6 months.
7.	1 : 2 ..	Lime : cinder	14 days and 6 months.
8.	1 : 3 ..	Lime : cinder	28 days and 6 months.

Studies on compression, absorption and flexural strength tests have also been conducted.

WORK DONE IN COLLABORATION WITH THE CENTRAL BUILDING RESEARCH INSTITUTE.

15. Under-reamed pile foundations.

The Central Building Research Institute has developed a new type of foundation design suitable for use in expansive clays. The bored piles are under-reamed to have a large bearing area and designed to take a greater load. To popularise and check up the efficiency of this design in various places, the Central Building Research Institute has offered to assist financially in putting up buildings adopting this type of foundation on swelling soils. A light building to house the Mobile ophthalmic van for the Erskine Hospital, Madurai, has been constructed. Necessary field instruments have been installed to measure the settlement etc., of this building. Data are being collected to study the efficacy of the design.

Financial Review.

A graph has been appended to this article which shows the extent to which the Central Government agencies like Central Board of Irrigation and Power, the Council of Scientific and Industrial Research, The National Buildings Organisation, etc., are financing the research schemes. These schemes, though of a fundamental and basic nature are useful to the whole country and they are of specific importance to this state as well in the solution of Engineering problems relating to this state. It may be worthwhile to point out that our laboratories have secured a pride of place among the various laboratories in the country and the research work done is being periodically reported in the various technical meetings and symposia conducted by the Central Board of Irrigation and Power, the National Buildings Organisation, etc.

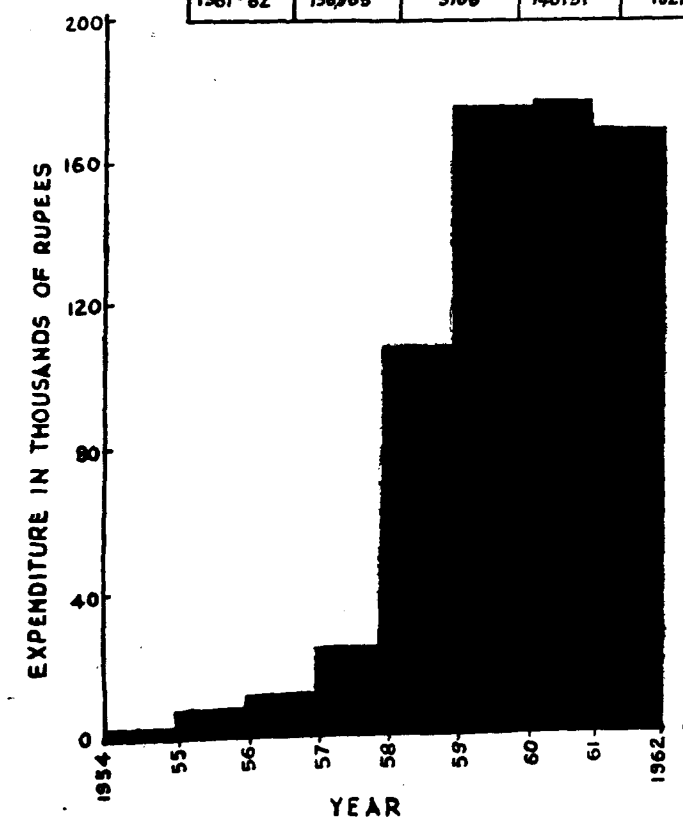
Though the laboratories were originally established with the specific object of solving problems relating to major irrigation works in this state, now that the major irrigation schemes have been completed, the emphasis of work in these laboratories must necessarily be on the fundamental and basic side, the results of which will have universal and wide application for the entire country and in this context, such centrally sponsored schemes are of great importance in the development of research activities in this state. It may be significant to mention here that the Central Board of Irrigation and Power and the Council of Scientific and Industrial Research have entrusted more number of problems to our research laboratories compared to those allotted to other state owned laboratories in the country which reflects largely on the trust and confidence reposed by the Central Board of Irrigation and Power and the Council of Scientific and Industrial Research in the high standard of work turned out all along. It will therefore be our endeavour in the years to come to seek more such centrally aided schemes and carry the research programme forward by utilising the entire grants made available to our research stations.

DESIGNS CIRCLE

UTILISATION OF CENTRAL FUNDS

EXPENDITURE — Rs

YEAR	C.B.I.P. SCHEMES		C.R.I.P.	C.S.I.R. SCHEMES		C.S.I.R.	N.B.O	1+2+3
	S.M&R. DN	OTHER SCHEMES IN POONDI	TOTAL 1	S.M&R.DN.	POONDI	TOTAL 2	3	
1944	IRRIGATION RESEARCH STATION AT POONDI ESTABLISHED							
1947	PHYSICS AND SOIL MECHANICS LABORATORY AT CHEPAUK ESTABLISHED. CONCRETE LABORATORY ESTABLISHED.							
1954-'55	—	—	—	3567	—	3567	—	3567
1955-'56	—	—	—	4520	4000	8520	—	8520
1956-'57	—	—	—	6302	5536	11838	—	11838
1957-'58	—	—	—	6922	19745	26667	—	26667
1958-'59	97,687	—	97687	8598	2219	10817	—	108704
1959-'60	168,564	—	168564	17123	2508	19631	—	188195
1960-'61	166,120	2088	168208	11973	4882	16855	3538	188601
1961-'62	136,965	3166	140131	18211	10280	28491	794	169416



NOTE :-

(1) THE EXPENDITURE FIGURE SHOWN INCLUDES THE RESEARCH LABORATORIES AT MADRAS AND POONDI

(2) THE FUNDS PROVIDED BY C.B.I.P., C.S.I.R AND N.B.O ARE PUT TOGETHER UNDER EXPENDITURE

FLOW OF WATER THROUGH SOILS

By

*Sri K. T. GANAPATHY, B.E. (Hons.).

1. Introduction.

The flow of water which takes place through soils is extremely slow as compared with flow through pipes and channels. Flow through soils is usually a stream lined flow, in which velocity is directly proportional to the hydraulic gradient. In order to establish relations in the realm of foundation and earth work engineering it has been necessary to investigate the physical properties of the different types of soils. Foremost among these are the permeability, the compressibility, the resistance against flow and shear characteristics of the soils.

2. Permeability of Soil.

A material is said to be permeable if it contains continuous voids. Since such voids are contained in all soils including the stiffest clays and in all non-metallic construction materials including sound granite and neat cement, all these materials are permeable. As water percolates through a permeable material, the individual water particles move along paths known as flow lines. If adjacent flow lines are straight and parallel, the flow is said to be laminar. Flow through soils is usually laminar and is expressed by Darcy's law.

$V = KI$ and $Q = KIA$.

where Q = Discharge in Unit time.

I = Hydraulic gradient.

A = Area of soil mass through which flow takes place.

K = Co-efficient of permeability.

'v' is defined as the quantity of water that percolates in Unit time across unit area of section oriented at right angles to the flow lines.

The channels through which the water particles travel in a mass of soil have a variable and irregular cross section. As a consequence, the real velocity of flow is extremely variable. However, the average rate of flow through such channels is governed by the same laws that determine the rate of flow through straight capillary tubes having a uniform cross section. If the cross section of the tube is circular, the velocity of flow increases according to Poiseulli's law, with the square of the diameter of the tube. Since the average diameter of the voids in soil at a given porosity increases practically in proportion of the grain size D. it is possible to express K the co-efficient of permeability on the basis of Poiseulli's law as

$$K = \text{constant} \times D^2$$

Allen Hazen obtained an empirical relation from his experiments on loose filters—

$$K\,\text{cm/sec} = C_1 D_{10}^2$$

D_{10} is the effective size in Cm.. C_1 in Cm/Sec varies from about 100 to 150. This relation is applicable to fairly uniform sands in loose state.

3. Limitations of Darcy's Law.

The limits governing the Darcy's laws depend on (*a*) Void ratio (*b*) The size of the Grains (*c*) The velocity and the gradient.

(*a*) When a soil is compressed or vibrated, the volume occupied by its solid constitutents remains practically unchanged, but the volume of the void decreases. As a consequence the permeability of the soil also decreases. In a soil that contains air bubbles. the size of the bubble decreases with increasing water pressure. The co-efficient of permeability of such soil increases with increasing hydraulic head. In clays with root holes or open cracks. percolation is almost inevitably associated with internal scour. The detached particles gradually clog the narrowest parts of the water passages

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whereupon the co-efficient of permeability decreases to a small fraction of its initial value. In all such cases the flow is not proportional to the gradient. Hence Darcy's law is not valid unless the volume and shape of the water passages are independent of pressure and time.

(*b*) According to Allen Hazen, flow through soils remains proportional to the gradient only if the effective size of the material is not more than 3 mm. This means Darcy's law is applicable to earth, sand and fine gravel and not to gravels the effective size of which is more than 3 mm.

(*c*) According to Professor Forchhimier even in coarse gravel the flow remains stream lined and not turbulent. He concluded that when the discharge is not proportional to the gradient. this is not due to turbulence but probably due to the pressure exerted by the filtering water upon the grains of which the material is composed. In very high velocity of flow the grains are more tightly pressed against each other and the permeability co-efficient does not remain constant. However in underground flow problems (i.e., percolation beneath hydraulic structures) such velocities seldom occur. Therefore in normal circumstances Darcy's law is generally applicable to percolation problem.

4. Seepage Computations.

FLOW NETS.

In order to compute the rate of flow of water through soils following Darcy's law it is necessary to determine the intensity and distribution of the neutral stresses, commonly known as the pore water pressure. These stresses can be determined by constructing a graph called flow net which represents the flow of water through an incompressible soil. An expression, known as Laplace's equation. governs the flow of every incompressible fluid through an incompressible porous material. When the flow is two dimensional the Laplace's equation can be expressed as—

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0$$

Graphically the equation can be represented by two sets of curves that intersect at right angle. The curves of one set are called flow lines, whereas the curves of the other set are known as equipotential lines. Every strip located between two adjacent flow lines is called a flow channel.

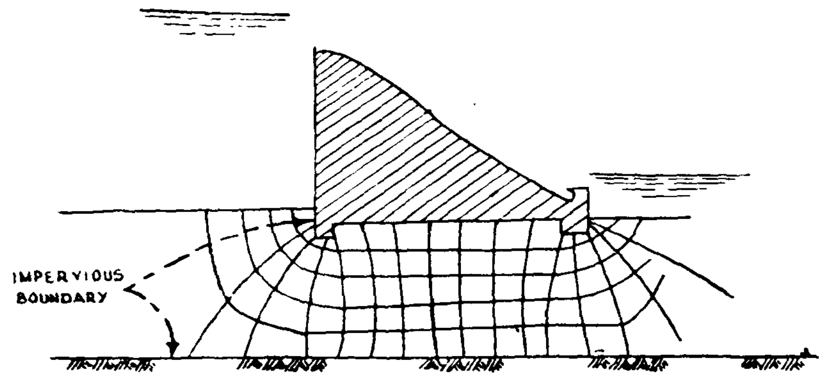
The flow lines can be plotted by (1) Graphical (2) Model and (3) Relaxation techniques.

Methods of construction of flow net.

1. GRAPHICAL METHOD.

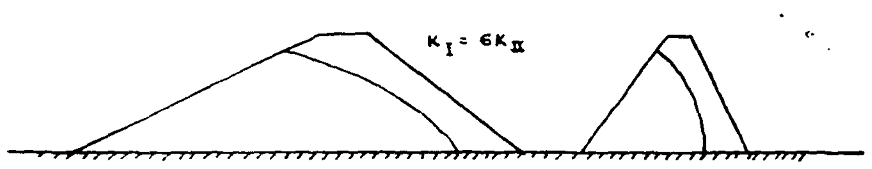
Before starting the construction of flow net, we must examine the hydraulic boundary conditions of the problem and ascertain their effect on the shape of the flow lines. The base of the dam represents the upper most boundary and the base of the pervious stratum represents the lowest boundary. The other flow lines lie between these two, and their shape must represent a gradual transition from one to the other. Further all the flow lines must be vertical when they meet the upstream and downstream ground surfaces. The first step in constructing the flow net is to draw several smooth curves representing flow lines that satisfy the requirements. Then several equipotential lines, which should intersect the flow lines at right angles, are drawn so that the fields are atleast roughly square. In this manner a first rough approximation to the flow net is obtained.

Taking the case of weir built on a pervious foundation with an impervious substratum at a certain depth below bed level, the upper flow line is drawn free hand following approximately the sectional outline of the apron. All sharp angles are however rounded off and the general shape of the line is therefore smooth. The space in between the line thus drawn and the apron is then divided into sections in such a way that the width of each section is approximately equal to its height. Under each of these original squares a new square is then built. The outer sides of the new squares will form the second line of flow. This process is repeated until we reach the bottom of the permeable layer. The last line must coincide with the rock surface. this serves to check the assumed alignment of the first curve. By this trial and error process the correct flow net diagram may be traced out (Sketch-1).



FLOW NET UNDER A WEIR ON PERVIOUS FOUNDATION

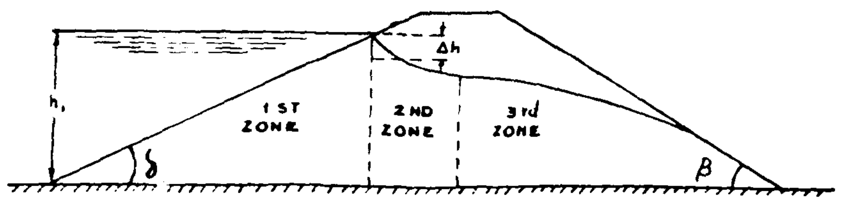
SKETCH. 1



TRANSFORMATION METHOD FOR ANALYSIS OF STRATIFIED SECTIONS

VERTICAL SCALE $\sqrt{\frac{K_{II}}{K_I}}$

SKETCH 2



SKETCH 3

Model Technique.

Two dimensional electrical analogy studies find extensive use in drawing flow nets for different sections. The basic principle that electric and hydraulic potentials are similar in nature is obvious. The phreatic line has to be fixed approximately by theoretical method, and equipotential lines are drawn with the help of null indicator and flow net patterns are got by drawing flow lines orthogonal to the equipotential lines.

Flow lines can also be traced in hydraulic models by using suitable dyes.

Relaxation technique can also be made use of for drawing flow nets.

5. Seepage Computation from Flow Nets.

It is convenient to construct the equipotential lines such that the difference between piezometric levels for any two adjacent equipotential lines is a constant. This difference is called the potential drop dh. If h_1 is the total hydraulic head and Nd is the number of potential drops.

$$dh = h_1/Nd$$

In order to derive the equations for computing seepage an area of side " a " is taken. The hydraulic gradient across the field is

$$i = \frac{dh}{a}$$

The discharge velocity

$$v = Ki = K.\frac{dh}{a} = K/a.\frac{h_1}{Nd}$$

If the width of the field measured at right angles to the flow lines is taken equal to an arbitrary value "b", the quantity of water that flows through the field, per unit width is

$$\Delta Q = bv = K.\ b/a.\ \frac{h_1}{Nd}$$

To simplify the computation of seepage, flow nets are constructed such that b = a, such that every field is square and

$$\Delta Q = \frac{Kah_1}{aNd} = \frac{Kh_1}{Nd}$$

If Nf is the total number of flow channels, this seepage Q per unit width is

$$Q = Nf\,\Delta Q = kh_1\frac{Nf}{Nd}$$

By means of this equation seepage can be computed readily after the flow net is constructed.

Every flow net is constructed on the assumption that the soil within a given stratum through which the water percolates is uniformly permeable. In nature every mass of soil is more or less stratified. The average permeability K_I in a direction parallel to the place of stratification is always greater than the average permeability K_{II} at right angles to the planes. To construct a flow net for such a stratified mass of soil, we substitute for the real δ soil a homogeneous material having horizontal and vertical permeabilities equal to K_I and K_{II} respectively.

The horizontal scale of the drawing is reduced by multiplying all horizontal directions by

$$\sqrt{K_{II}/K_I}.$$

For this transformed section we construct the flow net as if the medium were isotropic. The horizontal dimensions of the flow net are then increased by multiplying them by

$$\sqrt{K_I/K_{II}}$$

(sketch-2). The quantity of seepage is obtained by substituting the quantity

$$K = \sqrt{K_I \cdot K_{II}}$$

The expression for the quantity of seepage per unit width of the medium is

$$Q = h_1\frac{Nf}{Nd}\sqrt{K_I \cdot K_{II}}$$

6. Percolation in an Earthen Bank.

In every-day irrigation practice the percolation of water through an earthen dam is one of the important problems to be dealt with in detail. In seepage through an earth dam or dike, the upper boundary or upper most flow line, is not known, but must first be found. The upper boundary is a free water surface and is referred to as 'line of seepage' or phreatic line.

The first approximate mathematical solution for the quantity of seepage and the line of seepage through a homogeneous earth section on an impermeable base was proposed by Schaffernak, based on Dupuit's assumption that the average velocity in a vertical section was $v = q/h$ 'q' being the discharge per unit length and 'h' the depth of the filling stratum measured from the surface of saturation as the upper limit, down to the level of the impermeable sub-soil as the lower limit. The surface of the latter was assumed to be either level or inclined. This leads to a parabolic seepage curve.

$2\,qx = Kh^2$ which is often represented as

$2\,qL = K\,(h_2^2 - h_1^2)$. h_1 and h_2 are levels in two points of the saturation curve while L is the distance measured between them.

Dr. Dachler, in analysing seepage flow, divides the field into three zones (viz.), upstream and downstream slopes of the dam and the central intermediate part, the region where the hydraulic gradient is relatively flat. Particular attention is devoted to the upstream section for upon it depends primarily the seepage discharge percolating through the bank (sketch-3). The unit discharge Q is calculated from

$$Q = E_1\,K\,\triangle h$$

$\triangle h$ loss of head in the first zone. E_1 is a specific constant known as form factor—a co-efficient depending solely on the boundaries of the field. The form factor is expressed as

$$E_1 = 1{\cdot}12 + \frac{1{\cdot}93}{\cot \delta}$$

δ = slope of the bank

Seepage flow at the downstream face of the dam is also represented by empirical relation.

$$E = 0{\cdot}068\ \delta\ \left\{0{\cdot}86 + 0{\cdot}39\,\frac{\triangle h_3}{h_3} - \sqrt{\frac{\triangle h_3}{h_3} + 0{\cdot}36}\right\}$$

The ratio $\triangle h_3/h_3$ represents the relative loss of head in the third section. δ is the downstream slope. The scope of application of this equation is limited as it is valid for relative losses of head varying within 0.1 to 1.0. For the middle portion, Dupuit's equation is used.

$$E_2 = h_m.\ l.$$

h_m = average depth of permeable material.

l = length of that part.

Hence

$Q/K = E_1 \triangle h_1 = E_2 \triangle h_2 = E_3 \triangle h_3$

and $h = \triangle h_1 + \triangle h_2 + \triangle h_3$.

Arthur Casagrande in analysing seepage flow considers three cases depending on (1) less than 30 degrees. (2) greater than 30 degrees but less than 60 degrees and (3) greater than 60 degrees. The solution for β less than 30 degrees is based on the assumption that the equipotentials are vertical and the flow line horizontal. He gives the solution as

$$q = K.a \sin^2 \beta.$$

where 'a' is the wetted portion of the downstream face. For most cases encountered in earth dam design, the seepage may be calculated with sufficient precision by the equation

$$q = K\,(\sqrt{h^2 + d^2} - d)$$

The author has also given elaborate theoretical solution for the other two cases.

In this Station studies have been conducted on earth dams with hydraulic and electrical analogy models. The main aim of the studies is to see that validity of the theoretical assumption and it is found that the theoretical assumptions fairly agree with experimental values. A brief account of the experiments and the results thereof are given below.

7. (a) Seepage through Earth Dams.

The seepage through earth dams depends on many factors which include the type of material used and the method of construction of the dam, the type of foundation available, the depth and location of cut-off walls, the location and slope

of the internal core walls, and the drainage measures provided, etc. Experimental studies have been conducted to find out the effect of these various factors on the seepage quantity and the results are compared with the results obtained by electrical analogy method and with the computed values using a few of the well known formulae.

DIMENSIONAL ANALYSIS OF SEEPAGE FLOW.

The various factors affecting the seepage quantity per foot length of the dam are represented as

$$q = \phi\left(F, R, W, K_I, K_F, \frac{b}{H}, \frac{B}{H}, \frac{h}{H}, m_1, m_2, \frac{d}{H}, \propto, \frac{D}{H}, \frac{\beta}{H}\right)$$

R : Reynold's number.

W : Weber's number

F : Froude's number.

K_I : Permeability of the material of the dam.

K_F : Permeability of the foundation materials.

b : the top width of the dam.

H : the depth of water on the upstream side.

B : the bottom width of the dam.

h : the depth of tail water.

m_1 and m_2 are upstream and downstream slopes.

d : Depth of foundation.

$\propto$: factor representing stratification in foundation.

D : depth of cut-off.

β : distance of cut-off from the heel of the dam.

The effect of each variable in q was studied by putting up a number of hydraulic models. A hydraulic model of the standard homogeneous section of earth dam was moulded to conduct the experiments. The following are some of the salient features of the results:—

(*a*) the variation of h|H was studied along with theoretical formula of Dupuit's

$$q = \frac{K}{2l}(H^2 - h^2)$$

It was seen that this equation does not give a true picture of the seepage flow since in a certain range q increases with any increase in h instead of decreasing. The same can also be analysed by slightly modifying Kozeny's basic equation which is

$$q = 2 \propto a_0$$

$$\text{where } a_0 = \sqrt{d_1^2 + H^2} - d_1$$

Where d_1 = horizontal distance in the Casagrande's equation.

(*b*) the variation of q with m_1 was analysed along with Casagrande's equation

$$q = K\,a\,\sin^2 \propto.$$

$$\text{where } a = \sqrt{H^2 + d_1^2} - \sqrt{d_1^2 - H^2 \cot^2 \propto}.$$

$\propto$ = downstream toe angle.

The correlation between experimental and theoretical results is fairly satisfactory.

(*c*) The variation of q with b|H was studied and it is found that as the value of b|H increases the seepage gets reduced and the trend of variation seems to be more or less in a straight line with negative slope.

(*d*) The variation of q with d|D. It is concluded when a homogeneous dam is constructed over a pervious foundation, the cut-off wall does not have any appreciable effect on the quantity of seepage, unless there is an impervious core wall also.

8. Zoned Sections.

IMPERVIOUS CORE WALL ON PERVIOUS FOUNDATION.

Studies were conducted with Poondi red soil as vertical core wall in the model and the seepage quantity was found out. The slope of the core wall was gradually increased till it became 1 on 1 and the seepage quantity q was found out. The percentage reduction in seepage quantity is given as a ratio of

$$\frac{q_x - q_0}{q_0}$$

The percentage reduction of seepage was plotted against the slope of the core wall, and the equation of the curve was found to be

$$p\,Q = 43\,m^{0{\cdot}16}$$

pQ = Percentage reduction of seepage

m = slope of the core wall

It was considered that the depth of pervious foundation may have some effect on the results. So the experiment was repeated with a foundation depth of 90 feet. The equation was found as $pQ = 30m^{0{\cdot}16}$. It follows that only the multiplying factor varies and it is more for lesser foundation depth.

9. Core Wall of Finite Permeability on Impervious Foundation.

Studies were conducted to study the effect of core wall slope on seepage reduction for dams resting on level impervious foundation with core wall of finite permeability. The shell of the standard dam section was made of gravel of known sieve size and the core wall was made of sand of different permeability ratios. The equation of such curves was found to follow.

$$p\,Q = C_1\,m^{C_2}.$$

The value of C_2 is found to be about 0.42 and that of C_1 varies with permeability ratios.

The equations correlating pQ and core wall slope for different permeability ratios were found out. The variation of C_1 with ratio of permeability μ between the shell and the core wall is found to be

$$C_1 = 1{\cdot}3\,\mu + 12{\cdot}6$$

So the general equation for the family of curves correlating pQ and m for different permeability ratios

$$p\,Q = (1{\cdot}3\,\mu + 12{\cdot}6)\,m^{0{\cdot}42}$$

10. Some of the conclusions drawn from hydraulic and electrical analogy models are summarised. (1) The Dupuit's equation for the variation of seepage quantity with the tail water head is not satisfactory, when compared with experimental values while Kozeny's basic equation with slight modification may yield better agreement. Casagrande's equation for seepage flow is fairly satisfactory. (2) Electrical method gives slightly higher values when the variation of seepage quantity was studied with d/H. This may be due to the assumption of the same phreatic line adopted from Casagrande's formula for all foundation depths. (3) Some empirical relations are brought out on studies conducted on zoned sections. For the zoned section models on impervious foundation it was found that the seepage quantity varies with the increase of the core wall slope. The seepage is maximum for a vertical core, decreases rapidly for increase in slopes at lower range, but rather slowly for increase in slopes at higher range. It is learnt that the percentage reduction of seepage due to a core wall slope is greater for higher slope values and for higher values of permeability ratio of the core wall and the shell.

LOAD TEST CONDUCTED ON THE BRICK ARCHED BRIDGE ACROSS RIVER PONNIAR AT MILE 187/4 OF MADRAS-BANGALORE ROAD, NEAR HOSUR BY THE HIGHWAYS RESEARCH STATION, MADRAS

By

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Most of the existing bridges on our Highways are of the arched type. The assessment of the strength of these bridges has become a necessity in recent years owing to the introduction of vehicles of improved design hauling heavy loads at great speeds unknown in the days when these bridges were designed and put up. The assessment will help firstly in utilising the existing bridges, thereby effecting enormous economy and secondly in instilling confidence in the minds of the road user and the road maker by establishing the degree of safety in using these old bridges to modern conditions of traffic.

2. Mathematical Approach.

The evaluation of the strength of the existing arch bridges can be done either by mathematical analysis or by actual load tests. The mathematical approach suffers from too many assumptions that restrict the scope of an exact evaluation and in many cases, also from lack of design data of the bridges some of which are even hundred years old. Constructional details which contribute enormous strength to the primary arch structure vary from bridge to bridge and in most bridges, these details could be got subsequent to demolition only. Even if these are known, it may be extremely difficult if not impossible to include them in the mathematical approach.

Arched Bridge structures as discussed are therefore too complex and their component materials also too variable and of unknown character for their strength to be determined mathematically with any reasonable degree of accuracy. Only actual load tests on particular bridges can form a truer guide to the strength of similar bridges.

3. Object of the Test at Hosur.

The test conducted at Hosur by the Highways Research Station is the second in their series on arched bridges, the first one being that on the abandoned arch bridge across Uppodai near Sathur in Madras-Cape Road. The first test confirmed the British criteria that an arch bridge is safe for a 40 ton bogey if its deflection under a 20 T axle does not exceed 0.05 in. Valuable information regarding abutment movement, dispersion through fill, transverse strength of arch and the strength contributions of fill, parapet and spandrel walls were obtained. The test at Hosur had its object in getting some more light on these aspects and getting further information regarding British Criteria before it could be adopted in general.

4. The Bridge under Test.

The bridge (Fig. 1) consisted of three spans with a central span of 45 feet and end spans of 37 feet 6 inches. The arches were all three centred with a rise of ¼ span. The bridge was entirely built with brick in lime mortar. It was constructed 136 years ago.

5. Nature of Tests.

The tests conducted could be classified as (i) Superficial tests with vehicle loading and (ii) Detailed tests with reaction loading. The superficial tests consist of Load Distribution tests and Dynamic tests, the former give an idea

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of the way in which concentrated loads of various magnitude are distributed laterally and longitudinally over the arch and the latter of the effect of speed and impact on the strains and deflections in the structure.

5. 1. Load Distribution Tests.

The tests were conducted by placing the rear wheels of a pneumatic tyred type Mack truck of 20 T axle loading capacity in the following five dispositions at 1/4, 1/2 and 3/4 span lines of the central span.

(a) Truck near the left parapet

(b) Truck at 2 feet from the left parapet

(c) Truck at the centre line of the bridge

(d) Truck at 2 feet from the right parapet and

(e) Truck near the right parapet.

For each disposition, the deflections were measured at five points across each of the 1/8th, 1/4th. 3/8th, 1/2. 5/8th, 3/4th and 7/8th lines of Central span and at three points across every 1/4th, 1/2 and 3/4th lines of each of the end spans. The strains were measured only in the Central span. One inch SR. 4 strain gauges and three-inch Philips concrete strain gauges were used. Strains along and across the span were measured. In measuring the strains. Baldwin lima strain Indicator was used. The tests were conducted for four different loadings of the truck, namely, 5T. 10T, 15T and 20T.

In addition to strains and deflections. the spread of the piers and abutments at springing level and rotation of the same were also measured by using deflecto meters.

5. 1.2. Dynamic Tests.

The speed tests were conducted by running the vehicle at different speeds and noting the deflections when the vehicle crosses 1/4th, crown and 3/4th span positions of the central span. The strains in the crown of the Central span were also recorded through a circuit consisting of Philips strain pick up, Philips direct reading measuring bridge and an oscilloscope attached with an automatic recording camera (Fig. 2).

The impact tests were conducted by making the vehicle jump over 1/2 inch and 1 inch diameter rods laid across at quarter and mid lines of the central span. The tests were repeated with a Road Roller of 10 T capacity.

5. 2. Detailed Tests.

These tests were conducted with the idea of gathering data about the behaviour of the arch under concentrated loads and about the strength contribution of the various components of the arch such as parapet, pavement, spandrel filling and spandrel walls to the arch ring. For the object in view the test sequence was suitably planned in such a way that the arch can be loaded at the crown at the various stages of demolition with low loads. The tests were made for the following twelve conditions. in seriatum, the load being applied centrally in a line at the crown of the central arch by means of a Reaction system (Fig. 3):—

(a) test with parapet and pavement in all spans

(b) test without parapet in the Central span

(c) test without parapet in all spans

(d) test without parapet in all spans and without pavement in the Central span

(e) test without parapet and pavement in all spans

(f) test without parapet and pavement in all spans and without spandrel fill in the Central span

(g) test without parapet and pavement in all spans and spandrel fill in the Central and the end span (Madras side)

(h) test without parapet, pavement and spandrel fill in all the spans

(i) test without parapet, pavement and spandrel fill in all the spans and spandrel wall in the Central span

(j) test without parapet, pavement and spandrel fill in all the spans and spandrel wall in the Central span and end span (Madras side).

(k) test without parapet, pavement, spandrel fill and spandrel wall in all the spans

(l) Ultimate test to destruction of the Central span.

For the first eleven tests, the loads were increased by increments till the tension crack in the crown section travelled 19 inches upwards on the u/s face of the arch (24 inches thick) near the key stone and the measurement of strains, deflections. spread and rotation were made as was done during the superficial tests. For the ultimate test, as the stagings had to be removed, electrical strain measurements alone were made and that too only till elastic limit was reached.

Planning.

Much planning is necessary for conducting such tests involving application of heavy loads exceeding 100 tons. In this particular case, oil jacks had to be changed to water jacks during the application of the loads over 50 tons. It is very necessary to observe perfect levels on both sides for the transmission of the loads. When the load was increased to 60 tons, the loading arrangements slipped due to twisting of the horizontal beam applying the load on the arch. Again, during the transmission of the load, the wooden packings cracked as they were not able to take up strain under such heavy loads. The arches were so elastic that they deflected by more than 6½ inches. The play of the ram of the jack was hardly 6 inches and hence packing had to be given during the experiment which were all not originally thought of. The Engineers guessed that the breaking load will be in the order of 60 tons by their experience at Uppodai but the ultimate load went up to 117 tons with the result the loading arrangements had to be increased to afford necessary reactions for the application of the loading on the arch. Here again the original forecast had to be revised and arrangements made to increase the loading.

Discussion of Test Results.

In the superficial tests, the first crack appeared at the crown at 15T and the maximum deflection for 20T axle was about 0·08 inch at 30T Gross load. This will be satisfactory for the plying of modern vehicles (i.e. 8T, axle load with about 20T Gross load).

The effective width of dispersion was obtained by dividing the sum of the deflections over the full width by the greatest deflection and it bore remarkable agreement with the time honoured method of arriving at the effective width on the assumption of 45 degrees dispersion of load through the fill down to the intrados of the arch.

The dynamic strains recorded in the *oscillogram* showed that they were very much in excess of the static strains for the corresponding loads.

Graphs depicting deflections along and across the axis of the bridge showed that the arch barrel behaves as an arched barrel rather than as a series of arch ribs or strips.

During the detailed tests, one significant fact emerged and that was that there was outward spread of the piers whenever the bridge was loaded and it increased with the increase in the load. Such movements, which are ignored in the elastic design, can have an important effect on the stresses in the arch barrel Norman Davey in his " Test on Road Bridges " has shown that the stresses due to the spread increase the stresses calculated in the normal manner by 50 per cent. Viewed in that light, the spread movements seem to be an important factor that will have to be taken while assessing the strength of arch bridges.

Another feature observed was the development of tensile cracks at the crown at relatively low load (say 15T or 18T). These cracks closed up completely on removal of the loads, showing the perfect elasticity of the arch system even after formation of cracks. The significance of this feature lies also in the fact that the development of tension at the intrados as indicated by the presence of a crack or open joint does not entail collapse of the bridge. The ultimate load (117T) carried was far in excess of that required to cause tension and hence it seems inappropriate to assess the carrying capacity, as is done in mathematical analysis, on the basis of the load that causes tension.

A further phenomenon observed was that of the continuity between the three arches. When the crown portion in the Central span deflected down, the crowns of the end spans went up. In the ultimate test when the hinges were formed in the Central span correspondingly hinges were formed in the adjacent spans also, strictly satisfying the behaviour of continuous beams under load. This continuity was due to the very heavy and stiff haunch filling. Hence it seems appropriate to take into account the effect of continuity while assessing the strength of multi arch bridges.

Another notable point observed was that of the heaving up of the arch by ·06 inch when the spandrel fill (about 20 to 30T) in the Central span was removed. For producing the same deflection of ·06 inch 18T live load was applied at the crown of the arch. These indicate how dead loading forms the major portion of the load in an arch structure and how by avoiding spandrel fill, the same arches can be made to take greater live loads.

The removal of the arch components like parapet, pavements, fill and spandrel wall greatly change the deflection values obtained for the same loads and therefore it was evident that they all contribute to the strength of the arch.

Conclusion.

The investigation proved that the strength of existing arch bridges is greater than what is obtained by the known methods of calculations. The tests indicated that:—

(i) considerable abutment movement takes place which is not so far taken into account in design calculations.

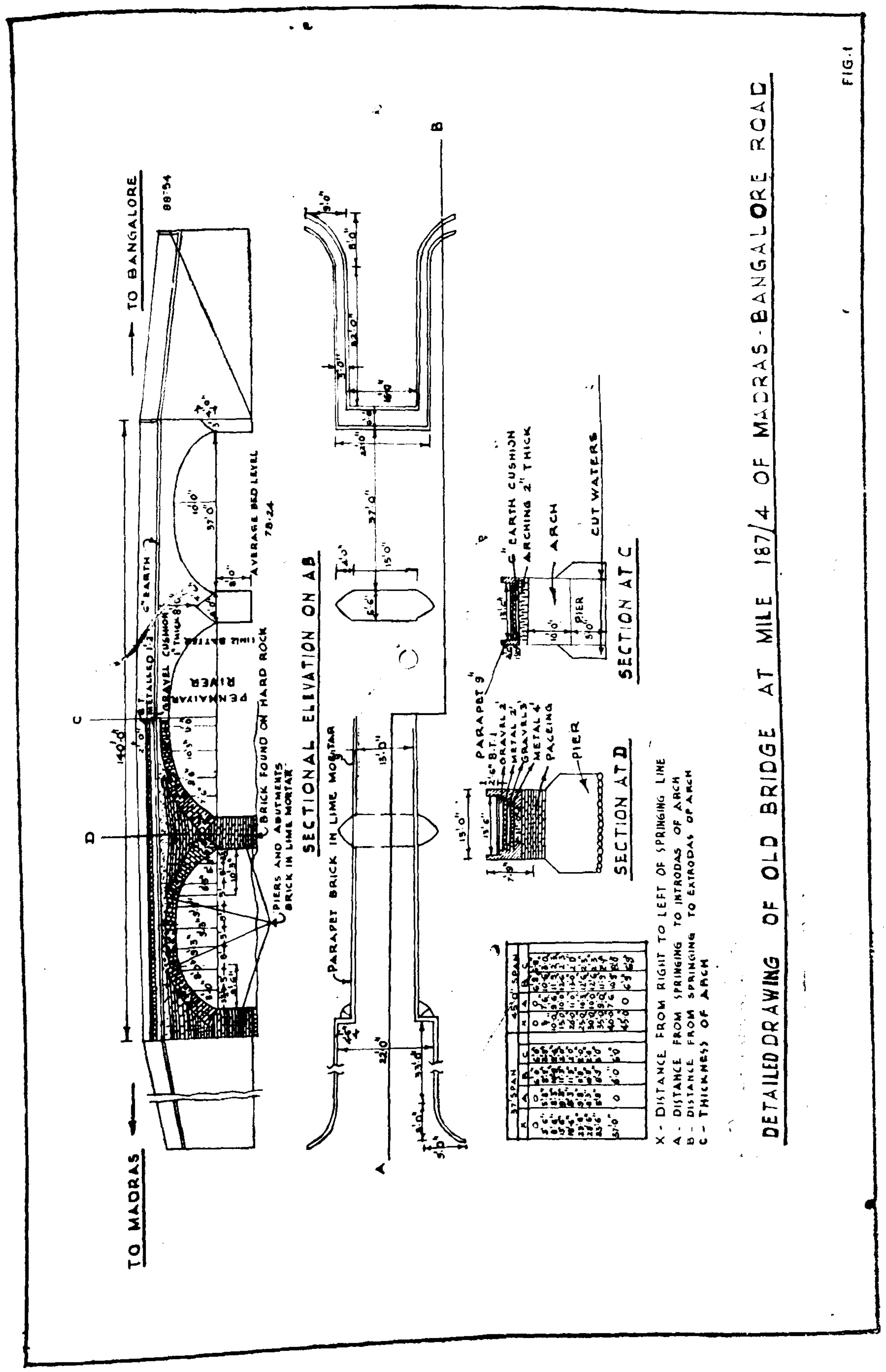
(ii) failure does not necessarily occur immediately after limiting tensile stresses are produced.

(iii) the arch barrel as a whole may behave as an arched slab rather than as a series of arch ribs or strips.

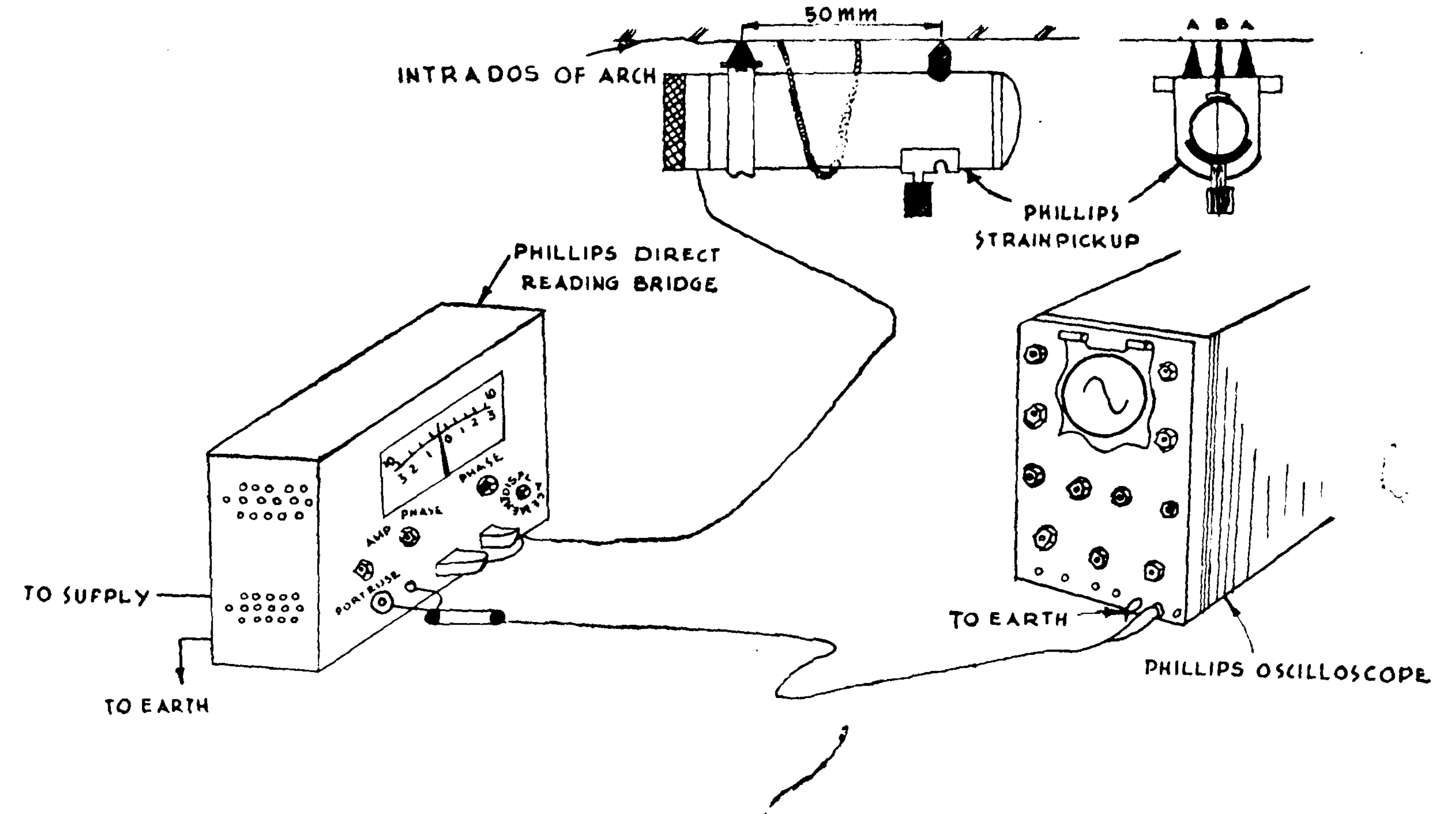
(iv) the components of the bridge over the arch contribute to the strength of the arch; and

(v) the elastic properties definitely exist even after the initial cracks have appeared (observance of initial cracks in a bridge may not be the ruling criteria for condemning it.)

The importance of these tests was fully recognised by the Government of India and they had sanctioned an amount of Rs. 20,000 for these tests. Similar tests could be undertaken by the Research wing of the Public Works Department on old abandoned buildings to be dismantled or on proto-types of portals, slabs and other framed structures.

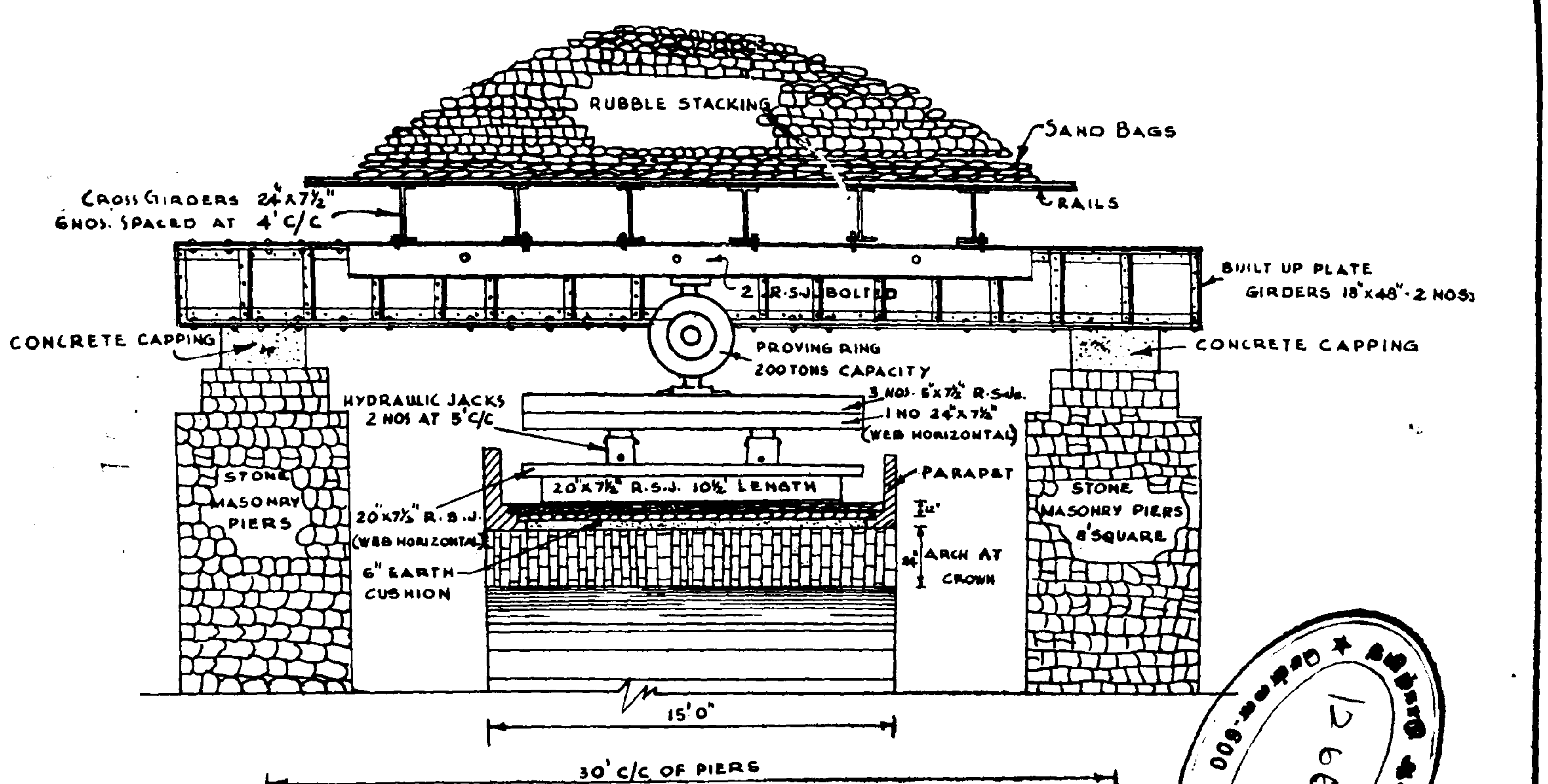


DETAILED DRAWING OF OLD BRIDGE AT MILE 187/4 OF MADRAS-BANGALORE ROAD



ASSEMBLY OF PHILLIPS STRAINPICKUP DIRECT READING BRIDGE AND OSCILLOSCOPE

FIG. 2

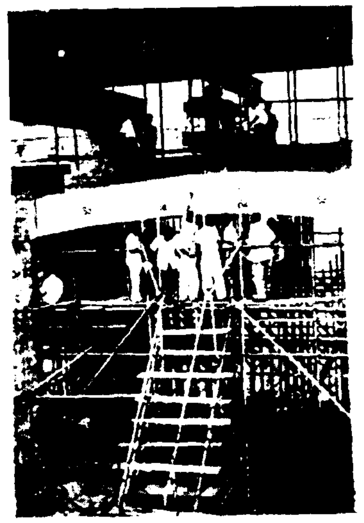


TRANSVERSE SECTION AT THE CROWN OF THE CENTRAL ARCH SHOWING THE LOADING ARRANGEMENTS

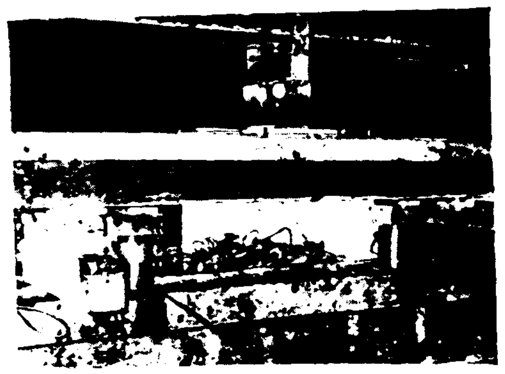
FIG. 3

1. *The Bridge under test.*

2. *View on top of Arch—filling removed.*

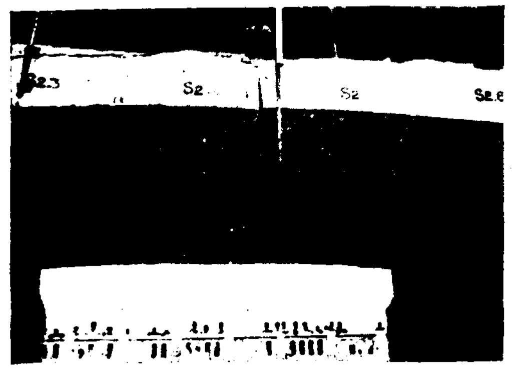


3. *Observing First Crack.*

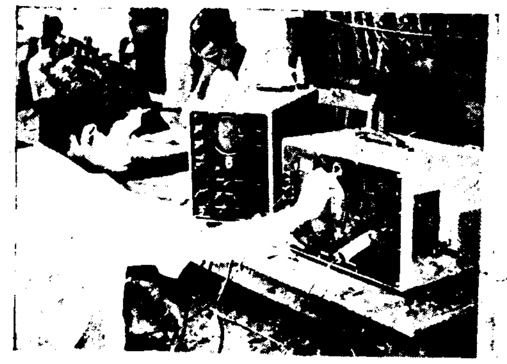


4. *Loading Arrangement.*

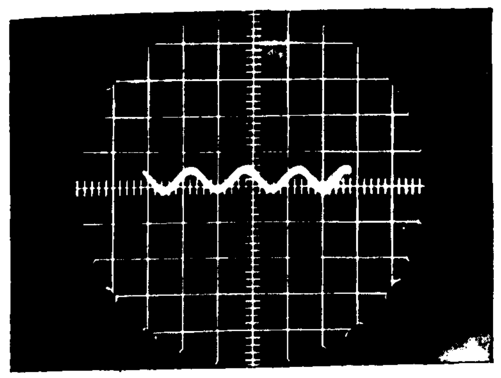
5. *View showing distress at ¼ Span.*



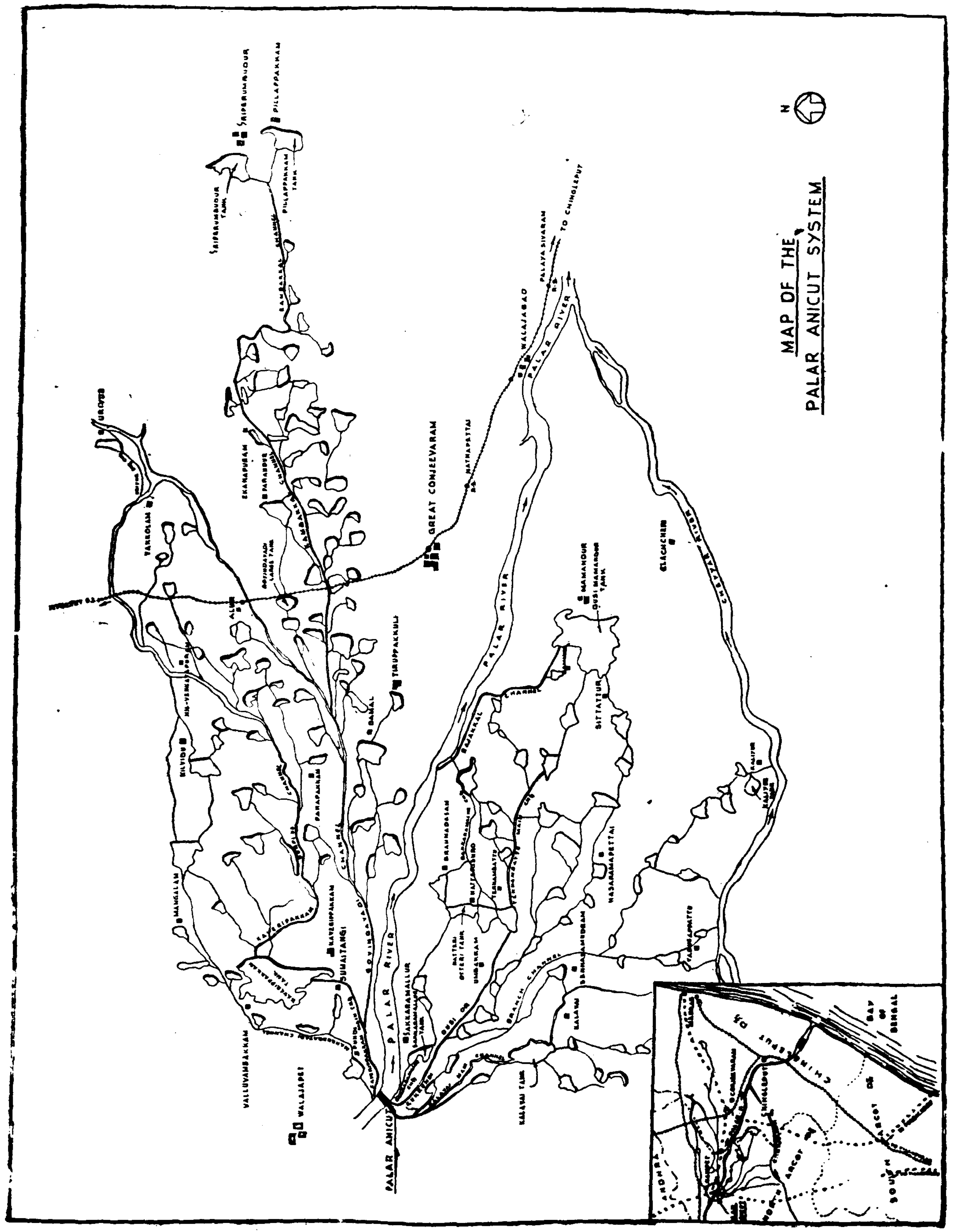
6. *Arch which has become flat before failure.*



7. *Electronic Instruments used for recording dynamic strains.*



8. *Oscillogram showing a recorded strain.*



THE PALAR ANICUT SYSTEM

BY

† SRI A. MOHANAKRISHNAN, B.E., M.SC.

The Palar River.

The River Palar has its source near Chintamani, in the Kolar District of Mysore State and enters the Madras State limits near Vaniyambadi. Of its total length of about 220 miles, the river runs for 150 miles in the Madras State. Two main tributaries to the River are the Ponnai and the Cheyyar. Ponnai joins the river Palar near Tiruvalam, North West of Ranipet Town and Cheyyar, south of Conjeevaram Town. Flowing down the Town of Chingleput, the river falls into the Bay of Bengal near Sadras.

Ancient Utilisation.

Palar waters have been efficiently utilised for tank irrigation from ancient times. Several supply channels with open heads have been dug from it on either side connecting chains of tanks formed to feed the lands. The tanks besides receiving supplies from their own independent catchments, had the monsoon run off of the river diverted through these supply channels and thus were assured of one or two fillings. Particular mention may be made of the two ancient tanks Mahendravady and Kaveripauk in the Palar Anicut System. Mahendravady is believed to have been in existence even in the 1st half of the 7th Century A.D. Records show that the Kaveripauk tank was constructed by the Pallava King Nandivarman III between the years 710-757 A.D. A supply channel was cut from the River Palar to this tank with its open head somewhere near the present Palar Anicut.

Besides these, the innumerable spring channels run from either bank of the River throughout its course, carrying the rich springs of Palar to irrigate sizable ayacuts by direct irrigation. These have been existent from immemorial days and have been maintained under Kudimaramath System.

Lift irrigation has also been in practice. The subsoil flows have been tapped driving large diameter wells on either flank in the basin which gave copious supply.

The Palar Anicut.

With a view to stabilise the ayacut receiving supplies through the several tanks and to remedy the unsatisfactory arrangements of forming Korambus for diverting the river waters through the supply channels, building of an anicut across Palar, 4½ miles below Arcot town was conceived and the scheme was executed between the years 1855 and 1858. The anicut has since been subjected to several flood damages, the worst having occurred during the floods of 1903 when the water level rose to the record level of 7.85 over the anicut crest and breached a portion which was restored in 1905.

The anicut is 2,628 feet in length and is built of brick in lime with a cutstone crest founded over two rows of wells sunk in the sandy bed protected with aprons on the upstream and down stream sides. There are nineteen scour vents on the left flank, nine of size 5 feet by 3 feet 6 inches and ten of size 5 feet by 4 feet and twenty of size 5 feet by 4 feet on the right flank. Portions of the anicut measuring 300 feet have been provided with 2 feet falling shutters on the crest.

There are four channels taking off from the anicut two on either side with their head sluices built in continuation of the anicut.

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Their particulars are as below:—

Name of channel.	Length of channel.	Sill of head sluice.	Number and size of vent.	Designed carrying capacity in cusecs.	Number of tanks fed.	Designed ayacut in acres.
(1)	(2) M. F.	(3)	(4)	(5)	(6)	(7)
Left side channels.—						
Mahendravadi	24 7	488·18	3 Nos. 5′ × 5′	552	28	4,849
Kaveripauk	10 7	488·18	18 Nos. 5′ × 5′	3,443	159	51,059
Right side channels.—						
Sakkaramallur	20 3	489·33	3 Nos. 6′ × 5′	624	21	5,105
Dusi or Tennambattu	23 1	490·18	15 Nos. 5′ × 5′	1,598	109	19,903
					317	80,916

Besides the above 317 tanks in the system which are directly fed by the supply channels, there are several others in the Chingleput District which benefit at least indirectly by the surplusing of the chain of tanks one of which is the large Chembarambakkam Tank. The surplus course of the Kaveripauk tank is itself the River Kortalavar in the lower reaches which feeds the Madras water supply system. The Palar Anicut System, thus benefits large portions of the North Arcot and Chingleput districts in the State where perennial river sources are non-existent. A system map is appended.

Existing Conditions and the need for improvement.

The benefits of the well conceived century old anicut system have been progressively on the vane and needed attention.

The flow days in the River have considerably diminished. Against the record of average of 87.5 flow days in a year during the period 1864-1879, the river is now practically dry throughout the year, save for a few flash floods of short durations. Added to this, the river bed has badly silted up, the head sluice vents largely choked and the channel beds also heavily silted for some distances down stream of the head sluices. Naturally the channels cannot draw their designed discharges even during the few days the river is in floods and thus a good part of the flows is lost surplusing over the anicut while the three hundred and odd tanks await in queue for their filling in vain. The tanks again have lost their capacities due to silting up year after year, the cultivated extent has been affected and over a period of 20 years the gap between the cultivated ayacut and registered ayacut has been found to be on average as much as 10,500 acres.

To partly bridge this gap in cultivation and to improve the efficiency of the anicut and channels the Government have sanctioned a scheme under the Third Five-Year Plan which is now in progress.

The Improvements Scheme.

The improvements envisaged under the scheme may be broadly mentioned under the heads.

(i) Improvements to the anicut;
(ii) Improvements to the channels; and
(iii) Improvements to the tanks.

All the improvements now proposed have been evolved after extensive studies in models at the Irrigation Research Station at Poondi to suit the conditions at site and with a view on economy.

Improvements to the Anicut.

(*a*) The silt in front of the head sluices on either flank would be removed and certain silt exclusion devices provided.

Divide walls would be built between the head sluices and the anicut scouring sluices which would so divide the flow in flood conditions and induce greater velocities on river side that the rolling silt would be mostly dragged in through the scouring sluices.

Grade walls with their top at the present river bed level would be built forming stilling pockets in front of the head sluices wherein the silt that escapes would be deposited and washed down to the riverside through the scour vents proposed to be provided in the divide wall.

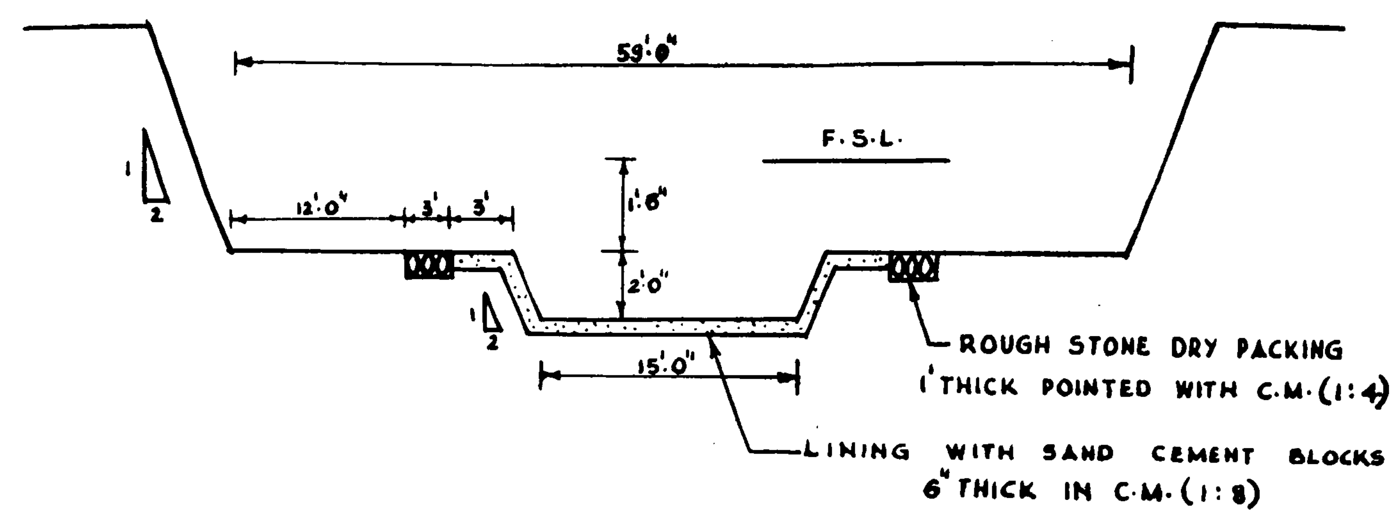
The sills of the scouring vents in the anicut which are now at the same level or higher than the sills of the head sluices in some cases would be lowered down to 2 feet below the sills of head sluices.

(*b*) Effective energy dissipation arrangements would be provided in rear of the anicut. There has been frequent damages to the rear aprons of the anicut during the past floods and recurring expenditure incurred on their restoration. It is now proposed to provide a 6 feet deep stilling basin in rear with a sloping floor to effectively dissipate the energy.

Improvements to the Channels.

Besides desilting the channels, there is need for evolving methods to draw off the maximum possible supplies whenever available. To achieve this a new type of design has been attempted. Two zoned section is proposed for these channels for the head reaches in place of the conventional trapezoidal section. The low flows would be confined to the inner section and at the time of floods maximum supply will be drawn over the berm section also. The inner section would be lined with sand cement blocks of mix 1 : 8 to withstand the higher velocities induced, the lining extending for a short width on the shoulders also. This two zoned section is adopted up to the first drop in the channel or upto the first tank only.

A typical two zoned section proposed for Mahendravady Channel is shown below. The design for the section which may be of interest is given below:—



Palar Anicut System
Proposed two Zoned Section of Mahendravadi Channel
(Not to Scale)

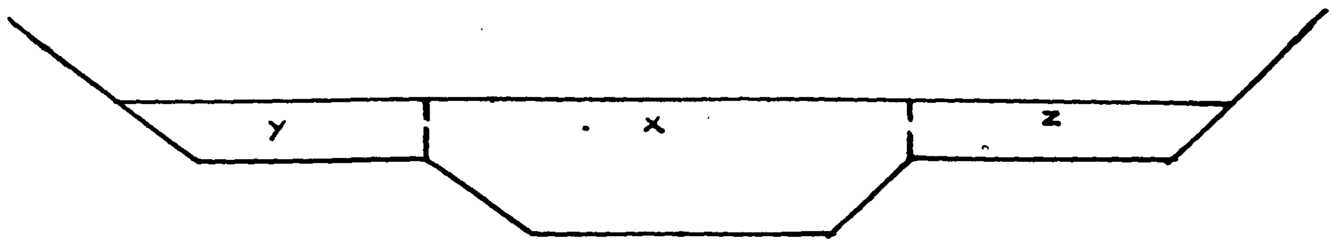
Data :

Designed carrying capacity (Original) ..	552 cusecs.
Low flows that should be carried in inner section. ..	110 cusecs.
Value of n, the co-efficient of rugosity for the channel sides. ..	0·025
Value of n, for the lined section ..	0·015
Bed slope S.	1 in 1230.
Full supply depth	3'6"

Design :

Inner Section

Assume a bed width of 15 ft. with side slopes. 2 to 1



Let the depth of inner section be .. 2'0"

Area A = 38 sq. ft.

Wetted perimeter P = 15+2 × 4.47 = 23.94

Hydraulic Radius $R = \frac{A}{P} = \frac{38}{23.94} = 1.588.$

Adopting Mannings formula,

$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$V = \frac{1.486}{0.015} \times (1.588)^{\frac{2}{3}} \left(\frac{1}{1230}\right)^{\frac{1}{2}}$$

V = 3.84 ft/Sec.

discharge Q = AV = 38 x 3.84 = 146 cusecs.

Two Zoned Section

Assume total width of 59 feet leaving a berm width of 18 feet on either side of the inner section.

Taking the central portion X

A = 38 + 23 × 1.5 = 72.5 sq. ft.

P = 23.94 ft.

$$R = \frac{A}{P} = \frac{72.5}{23\cdot94} = 3.028$$

$$V = \frac{1.486}{0.015} (3.028)^{\frac{2}{3}} \left(\frac{1}{1230}\right)^{\frac{1}{2}}$$

= 5.91 ft/Sec.

Q = AV = 72.5 × 5.91 or 429 cusecs.

Taking the side berm Y

A = 19.5 × 1.5 = 29.3. sq. ft.

P = 21.3 ft.

$$R = \frac{A}{P} = \frac{29.3}{21\cdot3} = 1\cdot376$$

$$V = \frac{1.486}{0.025} (1.376)^{\frac{2}{3}} \left(\frac{1}{1230}\right)^{\frac{1}{2}}$$

= 2·10 ft/sec.

Q = AV = 29.3 × 2.10 = 61.5 cusecs.

Discharge in berm Z = 61.5 cusecs.

Total discharge in the section = 429 + 61.5 + 61.5
= 552 cusecs.

Improvemen's to the Tanks.

The storage capacity of some of the selected tanks under the system would be at least partly restored by desilting-*cum*-reclamation, or raising F.T.L. etc., and the most needed repairs on them for the bund, sluices and the weir would be attended to.

Conclusion.

With these improvements effected it is hoped that the century old system showing signs of languishing slowly would have been rejuvenated and re-equipped to serve the needs of these areas for a long time to come. An average gap of 3,285 acres will. it is expected. be bridged thereby. The cost of the scheme is Rs. 48. 10 lakhs.

FLY-ASH AS A CONSTRUCTION MATERIAL

By

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Sri C. Raghavendra Rao, B.Sc. (Eng.) † and
Sri R. Krishnamurhy, B.Sc. ‡

1. Introduction.

Since independence various multipurpose projects and huge construction works have been undertaken in our country and many have been completed successfully during the First and Second Five Year Plan periods. Encouraged by the success of the two plans, a much more ambitious programme of construction has been undertaken during this Third Plan and a sum of about Rs. 2.000 crores has been allotted for this purpose. About 70 per cent of the total constructional costs consist of expenditure on construction materials. If the plan is to be implemented economically and successfully it is necessary to focus the attention of Engineers on two important aspects, viz., (1) stepping up the production of conventional building materials like cement, lime, brick, etc., and (2) finding out ways and means with the aim of reducing the cost of construction by improved practices. Moreover, the present programme of rapid industrialisation and construction of development projects under the plans have resulted in shortage of cement due to production falling short of demand. Therefore it is essential to develop and bring into use newer and comparatively cheaper materials in the field of construction.

The vast industrial development taking place in the country has resulted in increasing the quantity and variety of industrial wastes such as cinder, blast furnace slag, fly-ash, etc. Some of these wastes can be successfully and economically used as constructional materials. One such product is the fly-ash or the pulverized fuel as The use of powdered coal in thermal pow stations produces huge quantities of ash in th form of fine dust. This ash was once considere as a nuisance product. But now this is bein used in many fields such as manufacture of cla building bricks, foundry cores and sand blastin etc. It has also been found that fly-ash can b used as a pozzolona and as a part replaceme to cement resulting in cutting down the con truction costs.

Though the use of fly-ash in construction dams and other concrete structures has bee widely employed in many of the technical advanced countries, its use in large scale con ruction works does not appear to have gaine importance in India except in a few places lil Rihand. Hungry Horse Dam in the Unite States, Lednock Dam in Britain are a few pr jects where fly-ash has been extensively used combination with cement. Hence great attention should be paid to exploit and use th fly-ash as a construction material. This coul be achieved only if the Engineers make bold recommend the use of the same, especially mass concrete construction like concrete dan and flooring work for vast Industrial set up after a careful and thorough testing and stu on the quality of the fly-ash to be used as partial replacement to cement.

Detailed study has been conducted in th concrete laboratory on Basin Bridge fly-ash and preliminary study is under progress with the fl ash obtained from Neyveli Lignite Corporatio

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A short account of the properties of fly-ash, the tests conducted and the test results and the use of fly-ash in different fields are described in the following paragraphs.

2. Fly-ash and its Pozzolanic Action.

Fly-ash is the residue obtained as a result of burning pulverized coal and is collected as a waste material from Thermal Power Plants. It is grey in colour which generally depends upon the carbon content in the ash. Generally fly-ashes are much finer than Portland Cements. The principal constituents of fly-ash are silica alumina and iron oxide with varying amounts of carbon.

Though fly-ash does not possess any cementing property by itself and hence cannot be used as such independently, the inherent advantages derived by its usage in combination with lime or cement are well known. Fly-ash, when used with cement reacts with the free lime or amorphous calcium oxide, produced as a result of hydration and hydrolysis of cement compound at ordinary temperatures in the presence of water, to form stable insoluble compounds possessing cementing properties. Such a reaction is termed as pozzolanic action and the material used as a pozzolana. The use of pozzolana in combination with cement, also imparts to the resulting mortar and concrete some desirable qualities like workability, increased water tightness, reduction in alkali-aggregate reaction, etc., and above all the economy effected by substituting a part of cement thus reducing the quantity of cement required on the job. The properties of fly-ash depend to a large extent upon fineness and the carbon content which may vary depending upon the nature of coal and the degree and efficiency of its combustion and the method of collection of the ash. It has also to be borne in mind that not all fly-ashes may be suitable for use in concrete. Fly-ashes of low carbon content and high fineness values only have generally been used in concrete. It is also necessary that thorough investigation must be made before advocating a fly-ash from a particular source for use.

3. Tests conducted on fly-ashes collected at Basin Bridge Power House, Madras and at Neyveli.

A detailed study has been conducted with the Basin Bridge fly-ash to study the following properties of mortar and concrete:—

(*a*) Specific gravity and fineness and chemical analysis.

(*b*) Lime reactivity.

(*c*) Sieve analysis.

(*d*) Consistency and setting time.

(*e*) Tensile strength.

(*f*) Flexural strength.

(*g*) Compressive strength.

(*h*) Permeability.

(*i*) Heat of Hydration.

(*j*) Strength with age.

The results of the tests conducted indicate the possibility of the Basin Bridge fly-ash being a good pozzolana and it can be used as a part replacement or addition to cement for construction works up to about 15 per cent safely.

At present a detailed study has also been taken up with the fly-ash brought from Neyveli. This ash is being thrown out as a waste material from the Neyveli Power Station commissioned recently. Tests for chemical analysis, fineness, sieve analysis and lime reactivity have been completed. Though the tests conducted so far on this ash are more of a preliminary nature, the results obtained seem to be encouraging. The carbon content in the ash is very low and the lime reactivity strength obtained is 1,054 psi against the minimum of 600 psi as per Indian Standard Specification requirements for a pozzolana.

The Neyveli fly-ash satisfies the R_2O_3 percentage. But there is one noticeable difference in the chemical composition between the Neyveli fly-ash and other fly-ashes. The CaO content in the Neyveli fly-ash is comparatively very high when compared to Basin Bridge fly-ash. The CaO content of former is 25·3 per cent while

the latter it is 1.67. This high calcium content is perhaps due to lignite being used instead of coal. How far this may affect the properties of mortar and concrete are under investigation. The CaO however forms part of the neutral agents in a fly-ash. But its full implication in chemical combinations has yet to be studied. A statement comparing the results obtained so far, after the investigation of these two fly-ashes, is enclosed.

It is generally known that chemical tests have no general value for assessing a pozzolana and can be used only as an initial sorting test. However, in a recent article on the "Variation in pozzolanic behaviour of fly-ashes" M/s. Vincent, Mateos and Davidson have attempted to correlate the strength of fly-ash mortars with chemical composition expressed in molar fractions and have obtained some interesting results. The mole fraction is obtained from the chemical composition, by expressing the weight per cent of each compound divided by the formula weight. A triangular chart has been suggested for the correlation by the authors and the chemical composition has been divided into three groups forming the three sides of the triangle as favourable compounds (SiO_2 + R_2O_3), neutral compounds (CaO + MgO) and unfavourable compounds (SO_3 + C). This concept is quite a noval one and it will be worth studying for all fly-ashes with the molar approach.

4. Discussion and Conclusion on the use of Fly-ash.

From the test results of Basin Bridge fly-ash it may be concluded that this can be used as replacements for cement in construction works up to about 15 per cent quite safely. This estimate has been made with some caution even though the reduction in strength up to 20 per cent replacement of cement is not very much. The same conclusions have been drawn by the other laboratories (viz.), Koyna Laboratory, Koyna and Silt and Constructional Materials Directorate, New Delhi to whom samples of this fly-ash were sent for investigation. Moreover when the early strength of concrete is of prime importance the use of fly-ash has to be viewed with caution as the contribution of fly-ash to strength is manifested only at later ages.

So far, only surki or burnt clay powder has been more commonly used in mass construction works in India than any other type of pozzolana. The springing up of new thermal plants where pulverized coal is used as fuel will result in the production of enormous quantities of fly-ash as a waste by-product. This ash when brought into use will go a long way in offsetting the difficulties encountered due to the acute cement shortage now prevailing. Incidentally, the disposal of the waste has become a major problem for these big thermal power stations. Hence it will be a great blessing in disguise if this waste product can be consumed economically in construction works. In the present context of cement crisis, the use of fly-ash firstly prevents the collection of waste material which occupy large areas of useful land, secondly it reduces the cost of disposal which gets charged to the parent industry and thirdly it saves cement consumption without affecting adversely the strength and makes available, the cement thus saved for other works. The transportation charges will however have to be taken into consideration, though the use of fly-ash as a pozzolana, involves no production cost at all. Hence, wherever it is economically and easily available, the possibility of using the fly-ash available in the locality deserves consideration.

The use of fly-ash in mass concrete works and other works where it does not come into contact with steel has been widely prevalent. The effect of the fly-ash on the embedded steel in the reinforced concrete works has not been thoroughly investigated. It is generally feared that the presence of sulphur compounds in fly-ash especially as sulphides will be detrimental to the steel. This may not be so. From a review of the chemical analysis of fly-ashes of various sources, tested and tabulated by R. E. Davis, it will be found that the sulphur and sulphur compounds computed as SO_3 are well below 3 per cent. As the maximum percentage of sulphur is limited to 3 per cent and below, the amount of sulphur and its compounds is not materially different in the concrete whether fly-ash is used or not. Actually the amount of sulphate in fly-ash is about the same as the sulphate in cement itself. Hence, it stands to reason that if the concrete mix is designed to give the adequate strength required on the job, fly-ashes of low sulphur content, could be used without any hesitation. However, in reinforced concrete jobs fly-ashes have to be tested for the sulphate content.

An important consituent in the fly-ash that should not be lost sight of is the percentage of free carbon content. The maximum limit that is generally allowed is about 10 per cent. Within limits the percentage in the concrete becomes so small that it is well dispersed within the whole mass, and its effect upon the electrical conductivity of the concrete and consequently upon the corrosion of the steel becomes very minor. But its effect on the properties of concrete (viz.) strength, drying shrinkage and water requirement, will be more predominant if it exceeds the specified limit. The presence of carbon has to be viewed with caution because (1) it does not combine with the free lime released during the setting of cement, (2) it mostly exists in coarse fractions and acts as coarse inorganic material of low reactivity, (3) it is structurally weak and (4) it absorbs more water and in consequence the requirement of water for the concrete for the same workability is increased. The carbon has also lower specific gravity than the ash and cement.

When such are the bad effects of the high carbon content in the fly-ash the utilisation of the coal-ash, a waste product normally from steam locomotives, containing more than 12 per cent of free combustible carbon can be advocated only after a thorough investigation for its pozzolanic properties, especially strength and durability at longer ages. Another point against the use of coal-ash is its economics. The coal-ash as it occurs is very coarse and has necessarily to be ground to get the required fineness before use, as against the direct availability of fly-ash in fine powdered form involving no extra cost. The Madras Laboratories have in this connection tested the coal-ash obtained from the Railway Institute, Lucknow. It may be a national waste to burn a fuel partially without the maximum possible combustion. A high carbon content material can better be burnt as completely as possible as a fuel in a furnace instead of mixing with cement and used in construction jobs. Coal ash can however, be used on certain works.

The work done by the Madras State Electricity Board in employing the fly-ash in the manufacture of the R.C.C. poles for transmission of power will be worth reviewing here. The object of switching over from conventional steel and wooden poles to R.C.C. poles is two-fold, first being the shortage in production of steel poles, non-availability of wooden poles and second the reduction in the cost of each pole due to the part replacement of cement with the fly-ash in the concrete. In the manufacture of R.C.C. poles 30 per cent of fly-ash is replaced for cement and it is observed that poles containing fly-ash replacements give better strength when compared with poles that contain no fly-ash. The saving in the cost of R.C.C. poles by the adoption of fly-ash, works out to about Rs. 3.30 per pole for 30 per cent replacement of cement. By the use of fly-ash in R.C.C. poles it is considered that one of the main advantages will be that the cost of distribution of electric power can be eventually reduced. The bold step taken by the Madras State Electricity Board deserves appreciation.

On the same lines, if it is assumed that 15 per cent of cement had been replaced by the fly-ash in all the building constructional activities of the P.W.D. in this State, the savings that would have resulted will not only be considerable but also the cement thereby saved due to fly-ash replacement can be used on more building projects under the plan schemes. It is high time the Building and Irrigation Engineers make up their minds to use fly-ash. National economy demands the ability of scientists and Engineers to use locally available material to the best advantages in the development projects in building up the country's prosperity and standards. In Madras State itself there are two

big sources of supply, one from Basin Bridge Power Station and the other from Neyveli Power Station. Likewise, practically in every state there will be more and more thermal power stations throwing large quantities of fly-ash now termed as industrial wastes.

In addition to the use of fly-ash as a pozzolana replacing part of cement, economically it has been used in other fields also. The presence of unburnt carbon in the fly-ash has facilitated in the development of sintered light weight aggregates. This has been studied in the United Kingdom and other countries and now in India also, it is being studied at the Building Research Centre, Roorke. The study on the light weight aggregates is now mainly discussed on the academic and research basis only, and the light weight aggregates have not yet been used on jobs sufficiently to report on the same.

In recent years, fly-ash is being used in the manufacture of clay bricks by incorporating some percentage of fly-ash with the clay. The use of fly-ash in clay-bricks seems to facilitate in reducing the difficulties experienced with sticky clays and affords better burning probably due to the availability of unburnt carbon in the ash. Fly-ash has also been used for soil stabilisation in road making and as a fill material especially in the United Kingdom.

From the above it is clear that research studies on the various uses of fly-ash, especially in the case of major construction works are in full swing all over the country and bound to produce great benefits. Not only laboratory work has to be stepped up, but also prototype studies by actually venturing to use the material on various kinds of jobs are necessary. We should get over the traditional shyness, fear and suspicion in using this new material on construction jobs with the help of laboratory tests and recommendations. The laboratories in Madras and other States have done valuable work on the fly-ashes and hence construction Engineers may be assured of competent laboratory control in the use of fly-ashes.

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Number and nature of tests.	*Neyveli Fly-ash.*	*Basin Bridge Fly-ash.*
I. *Chemical Composition.*—		
Loss on ignition	0·81	4·41
Silica	35·63	60·32
Alumina	26·06	26·58
Iron Oxide	7·36	4·77
Calcium Oxide	25·30	1·67
Magnesium Oxide	3·97	1·18

Number and nature of tests.	*Neyveli Fly-ash.*	*Basin Bridge Fly-ash.*
II. *Sieve Analysis.*—		
Per cent Retained on A.S.T.M. 50 sieve	0·50	0·25
Do do. 100 sieve	5·25	2·25
Do. do. 170 sieve	12·88	7·00
Do. do. 200 sieve	17·88	10·50
III. Fineness by Blaine's apparatus in Sq. Cm/grm	3171	3681
IV. Lime reactivity, compressive strength in psi.	1054	514

"TAKE INTEREST, I IMPLORE YOU,
In those sacred dwellings
which one designates
By the expressive term
LABORATORIES.

Demand that they be multiplied,
That they be adorned.

These are the temples of the future ..
Temples of well-being and of happiness.

THERE IT IS THAT HUMANITY GROWS
GREATER, STRONGER, BETTER."

—*Pasteur*

(1)	(2)	(1)	(2)